Available online at www.sciencedirect.com
SciVerse ScienceDirect
Procedia Engineering 54 (2013) 207 – 221
Procedia Engineering
www.elsevier.com/locate/procedia

The 2nd International Conference on Rehabilitation and Maintenance in Civil Engineering

Seismic Assessment of a Full-Scale Double-Storey Residential House using Fragility Curve

Nor Hayati Abdul Hamid[a*] and Nor Mayuze Mohamad[a]

[a]*Faculty of Civil Engineering, Universiti Teknologi MARA, Selangor, Malaysia.*

Abstract

Seismic assessment of double-storey house is investigated using fragility curve. Fragility curve is employed to determine the percentage Confident Interval for a precast house based on the experimental work. A full-scale of precast house is constructed on strong floor and tested under quasi-static lateral cyclic loading. The seismic performances of two parallel walls are observed during experimental work and classification of their damage states are according to the drift limits. Visual observations on the structural damages are recorded such as width of cracks on the wall-column interface, crack propagations on the column, spalling and crushing of concrete. The damage states limit of these walls panels are according to the definitions and descriptions as given in HAZUS 99-SR2. Colour-coded system is fully utilized in order to identify performance level, damage level, drift damage and ductility factors. Fragility curve is developed based on the probabilistic hazard level, cumulative probability function and classification damage-states. Level Confident Interval safety of double-storey house is assessed based on the plotted fragility curve and experimental work. Prediction damage states of this house at Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) can obtain from fragility curve analysis.

Selection and peer-review under responsibility of Department of Civil Engineering, Sebelas Maret University

Keywords: colour-coded system; damage states limit; fragility curve; confident interval; design basis earthquake; maximum considered earthquake.

* Corresponding author.
E-mail address: norha454@salam.ultm.edu.my

Selection and peer-review under responsibility of Department of Civil Engineering, Sebelas Maret University
doi:10.1016/j.proeng.2013.03.019

1. Introduction

Frequent earthquakes in neighboring countries such as Sumatra, Java Island and Philippines had trigger some of semi-active and sleeping fault lines in Malaysia. Some of the semi-active fault lines in West Malaysia are Lebir Fault, Terengganu Fault, Bukit Tinggi Fault and Kuala Lumpur Fault, while in East Malaysia are Keningau Fault, Labou-Labou Fault and Tabir Fault. Past earthquakes records occurred in Banda Acheh, Pulau Nias and Padang, Sumatera ranging from 4.4 to 9.4 on Richter scale had significant impacts on RC structures in West Malaysia. Recent earthquake in Silboga, Sumatera with 5.6 Richter scale and epicenter of 481km from Kuala Lumpur causes some tremor to the people who live in high-rise buildings around Klang Valley, Melaka and Negeri Sembilan.

Local earthquakes in West Malaysia such as Bukit Tinggi, Dam Kenyir, Jeranatut and Manjong did not cause any severe structural damages to the reinforced concrete buildings within their vicinities. These earthquakes have very low magnitude ranging between 2.6 to 4.8 on Richter scale with low intensities. However, Peninsular Malaysia is not as prone to tremors, but over a period of three years beginning in 1984, the area around Kenyir Dam in Terengganu recorded about 20 tremors, the strongest of which registered at magnitude 4.8 on Richter scale. Bukit Tinggi in Pahang was hit by three earthquakes on Nov 30, 2007, followed by more than 10 separate events until the last in May 2008, but the strongest was at magnitude of 3.5 on the Richter scale. There were two more isolated earthquakes occurred in Manjung, Perak and Jerantut, Pahang, on April 29 and March 27 in year 2009, measuring at 3.2 and 2.6 Richter scale, respectively. However, these earthquakes did not cause any structural damages to RC buildings. Most of the RC buildings in Malaysia are designed in accordance to BS8110 where there is no provision for earthquake loading at all. The design load consists of dead load and imposed load which acting in vertical direction only.

Figure 1 shows the hazard map together with peak ground acceleration (PGA) for West Malaysia with return period of 500 years (exceedance 10% in 50 years under DBE). The highest range of PGA for West Malaysia is between 0.08g to 0.1g located along the West Coast of Selangor, Perak and Melaka. Figure 2 shows the hazard map for East Malaysia with PGA between 0.1g to 0.12g.

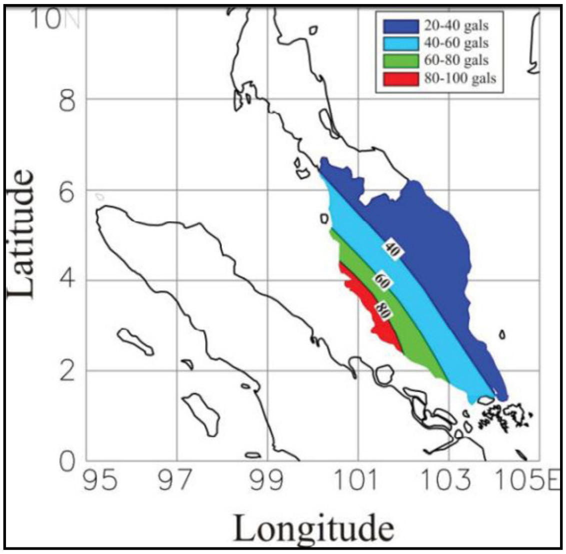


Figure 1. Hazard map for West Malaysia in 500 years (Azlan, 2010).

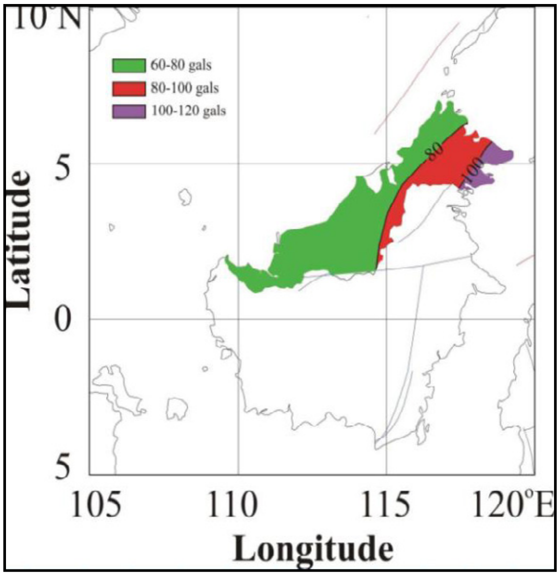


Figure 2. Hazard map for East Malaysia in 500 years (Azlan, 2010)

Figure 3 shows the hazard map of West Malaysia for the peak ground acceleration with return period of 2500 years (exceedance 2% in 50 years under MCE). The maximum value of PGA is situated in East Malaysia ranging between 0.18g to 0.12g as shown in Figure 4.

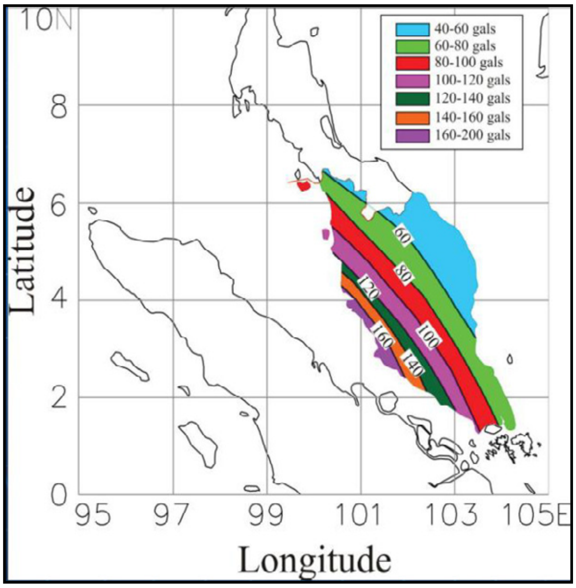


Figure 3. Hazard map for West Malaysia in 2500 years (Azlan, 2010)

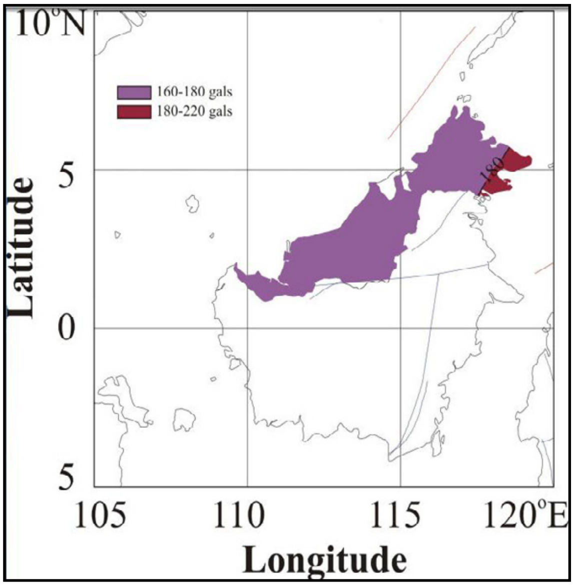


Figure 4. Hazard map for East Malaysia in 2500 years (Adnan, 2010)

Currently, the hazard map with different ranging peak ground acceleration is not adopted in the current code of practice. Consequently, most of the reinforced concrete buildings in Malaysia are designed according to BS 8110 where there is no provision for earthquake loading. The level of safety for these buildings is still questionable if bigger earthquakes occur either in Sumatra or near-field earthquakes. Therefore, the intention of this research is to determine the probability of survival double-storey residential house under DBE and MCE using fragility curve. This type of residential house is erected using precast shear-key walls and cast-in-situ concrete for the slabs, beams and columns. Fragility curve is utilized by incorporating the drift damage limit which obtains from experimental work.

2. Structure Finding from Previous Research

Relationship between peak ground acceleration (PGA) and structural damage is frequently used to estimate the distribution of structural damages in buildings over certain seismic regions. Blejwas and Bresler (1978) proposed damage states of structures can be measured by taking the ratio of demand on the seismic response over the capacity of the system. Meanwhile, Banonet al. (1981)defined damage state parameters in terms of rotation ductility, curvature ductility, flexural damage ratio (FDR) and normalized cumulative rotation (NCR). Later on, Banon and Veneziano (1982) pointed out the necessity to define the terms flexural damage ratio (FDR) and normalized cumulative ratio (NCR). They defined the flexural damage function (FDR) as the ratio of initial flexural stiffness to the reduced secant stiffness and normalized cumulative rotation (NCR) is the ratio of cumulative plastic rotations in n_{cycle} cycles to the yielding rotation of the nonlinear spring. However, Park and Ang (1985) expressed the seismic damage of reinforced concrete structures as a linear combination of maximum deformation and absorbed hysteretic energy. To prove this relationship, extensive damage analyses of Single Degree of Freedom (SDF) system and a typical Multi-Degree of Freedom (MDF) reinforced concrete building were performed. Theoretical results showed a simple relationship between the destructiveness of seismic ground motion in terms of characteristic intensity and structural damage in terms of Damage Index (DI).

Further study was conducted by Dis Pasquale and Cakmak (1990) on global damage indices for the complex structures using an optimal time variant linear model fitted to strong motion records. They discovered a good correlation between the numerical values of damage indices with actual visual observation of the structures. More explorations on building damage functions made by Kircher et al. (1997) for earthquake loss estimation using others parameters such as ground shaking characteristics, site/soil amplification and shaking durations. Further verification was made by O'Rourke and So (2000) on the seismic performance of 400 water tanks in nine separate earthquake events. They realized that the relative amounts of stored water contents in the tank and the tank's height to water tank diameter ratio had a significant influence on the tank's seismic performance.

After defining the parameters on structural damages, structural indices and loss estimation, another method is required to assess the probability of damages states in relation to ground motion. This method is known as fragility curve. Fragility curve can predict the probability of reaching or exceeding specific damage states for a given level of peak earthquake response. The probability of being in a particular state of damage and the input used to predict building-related losses are calculated by taking the difference of damage states in the fragility curve analysis. The expected seismic performance of the structures system can be achieved by combination the fragility curves, probability of ground shaking and an integrated possible outcome such as Monte-Carlo simulation Singhal and Kiremidjan (1996). One application of fragility curves was tested on gravity-type quay walls (Ichii 2004). He proposed design charts based on effective stress-based FEM and some parametric study on gravity-type quay walls. Then, fragility curves were generated by considering the difference between the observed displacements in case histories and estimated displacement by the chart. These proposed fragility curves are useful in assessing the restoration cost of the wall after an earthquake, real-time damage level and optimization of cost-benefit analysis under the requirement of seismic performance level. Until now, there is no study conducted in assessing the safety of double-storey house using fragility curve. Therefore, this paper focuses on the seismic assessment and safety of precast double-storey house under DBE and MCE.

3. Characterization of House's Damages

A full-scale of double-storey residential house was constructed at heavy structural laboratory and tested under reversible quasi-static lateral cyclic loading. Figure 5 shows the isometric view of double-storey house which has been constructed on strong floor in Jabatan Kerja Raya Heavy Structural Laboratory, Kuala Lumpur, Malaysia. The overall height of the building is 5000mm; length and width of foundation beam are 4.5m and 3.9m, respectively. WALL 1 and WALL 2 are constructed using shear-key precast wall system and the connections between these walls were made from cast-in-situ concrete and at the same time behaving as monolithic column which connected to the foundation beam. A total number of ten (10) linear potentiometers are used to measure the in-plane displacement/deformation when the applied reversible cyclic loading was imposed at in-plane direction of WALL 1 and WALL 2.

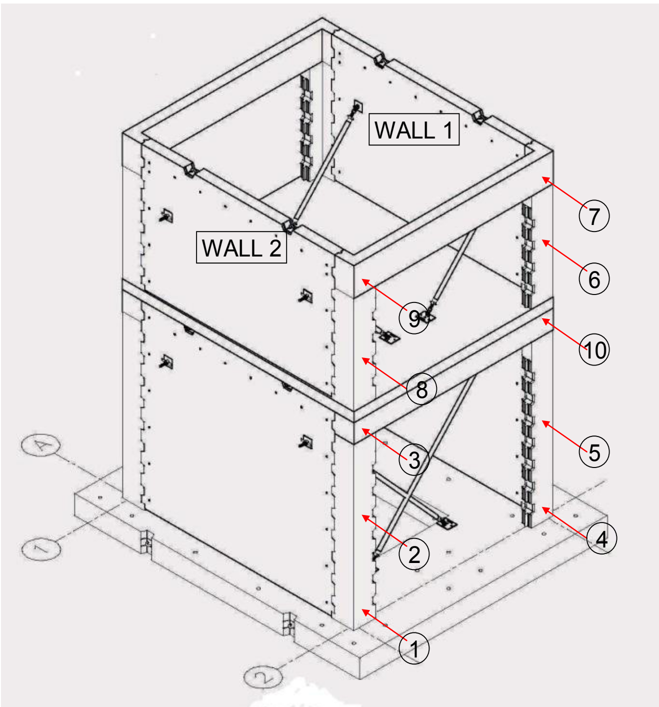
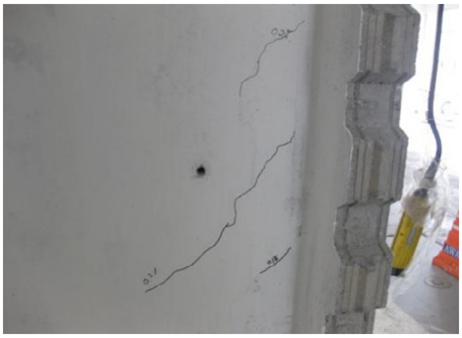
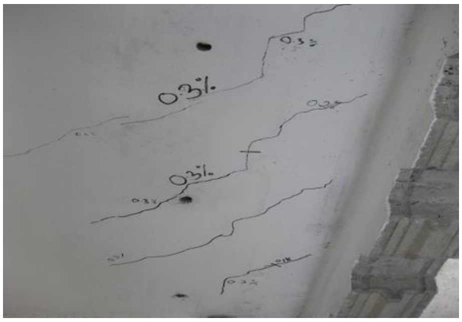
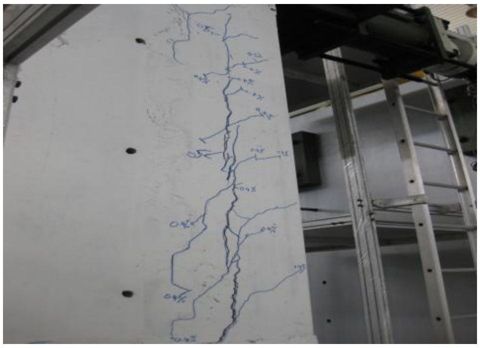


Figure 5. Isometric view of a double-storey house

Table 1 shows the visual damages during experimental work with respect with drift, damage state, description and ductility. WALL 1 and WALL 2 were pushed and pulled at ±0.05%, ±0.1%, ±0.2%, ±0.3%, ±0.4%, ±0.5%, ±0.6% and ±0.7% drift.

Table 1. Damage based on the visual observation during experimental work

Visual damages observed during experimental work	Drift	Damage State	Description of Damage	Ductility
	0.05%	1	No damage or crack was observed at any parts of the building. The building remain fully functional and can be occupied after the earthquake	$\mu = \frac{2.25}{13.5} = 0.17$
	0.1%	1	The building experienced only slight damage with few hairline cracks at the inner part at the connection on wall-column. The wall remains elastic and fully functional.	$\mu = \frac{4.5}{13.5} = 0.33$
	0.2%	2	The wall has slight structural damage such as wider cracks occurred on cast-in-situ column. The building experiences minor damage and it is remain functional after earthquake.	$\mu = \frac{9}{13.5} = 0.67$

	0.3%	2	The wall has minor damage with longitudinal cracks occurred between wall and column. Bigger cracks were observed at the inner wall panels. Minor cracks also notified on slab but the remain functional	$\mu = \frac{13.5}{13.5} = 1.00$
	0.4%	3	More cracks were observed at cast-in-situ columns and beams, crack on first floor slab, spalling of nominal concrete cover. Building losing its elastic stiffness and experienced significant damage.	$\mu = \frac{18}{13.5} = 1.33$
	0.5%	3	Wider opening of cracks at wet connections and more spalling of concrete. Strength degradation occurs with some lateral force remains and the building need to repaired before occupied.	$\mu = \frac{22.5}{13.5} = 1.67$
	0.6%	4	The building has lost a significant amount of its origin stiffness. A lot of cracks and spalling of concrete observed in the column, wall, beam and slab. The building is not safe.	$\mu = \frac{27}{13.5} = 2.00$

	0.7%	4	Building lost the stiffness and strength. Severe structural damage occurred at joint intersection of beam-column and column-wall interfaces. The building experienced near collapse phase.	$\mu = \frac{32.5}{13.5} = 2.41$

4. Seismic Assessment of the House using Fragility Curve

There are several models which can be used to quantify the damages, characterization of damage state and estimation of losses after the earthquakes. One of the models used in this research is called "HAZUS 99-SR2" which had been developed by Federal Emergency Management Agency (FEMA) and National Institute of Building Science (NIBS). The primary objective of HAZUS 99-SR2 (2004) is to provide a methodology and software application to develop earthquake losses on a regional scale. The loss estimation is useful for local, state and regional officials to plan and stimulate efforts in reducing risks from earthquakes and to prepare for emergency response or recovery. The loss estimation is based on the damage states of the buildings/structures after earthquakes. The status for damage states for the overall buildings after an earthquake is tabulated in Table 2. The combination of damages states as stated in Table 2 and Table 1 are used to generate Table 3. Table 3 is produced by tagging the colour-coding against the performance level, description of the damages, structural damage based on the percentage of drift and the upper limit of the ductility factor. Table 3 and Table 4 are merged together to produce a fragility curve for double-storey house using the following equations as mentioned in section 5 of this paper.

Table 2. Definition of damage states (HAZUS 99-SR2, 2004)

Damage State	HAZUS Descriptor	Post earthquake Utility of Structures	Evidence	Outage time	Expected Ductility Factor
1	None	No damage	None (pre-yield)	-	0.33
2	Slight	Slight damage	Cracking	< 3 days	1.0
3	Moderate	Repairable damage	Large cracks cover spalled	< 3 weeks	1.67
4	Heavy	Irreparable damage	Failure of components	< 3 months	2.0
5	Complete	Irreparable damage	Partial/total Collapse	> 3 months	2.7

Table 3. Definition of colour-coding and performance level for precast shear wall

Tag Colour	Performance Level	Description of damage level	Drift Damage	Ductility Factor
Green	Operational	Minor cracks, no damage, building occupiable	0.1%	0.33
Yellow	Functional	Wider cracks, initial spalling at corner of walls with moderate level of damage. The warehouses can be entered to remove belongings.	0.3%	1.00
Orange	Life Safety	Extensive spalling along bottom walls, longitudinal bars buckling with heavy damage on the walls. Warehouse can be entered for short periods for removing important items.	0.5%	1.67
Red	Near Collapse	Fracturing of longitudinal bars, no stability of structures, near collapse. The building cannot be entered.	0.7%	2.41

5. Theoretical Development of Fragility Curve

A fragility curve describes the probability of reaching or exceeding a damage state at a specified ground motion level. Thus, a fragility curve for a particular damage state is obtained by computing the conditional probabilities of reaching or exceeding that damage state at various levels of ground motion. The probabilistic hazard levels frequently used in FEMA 273 (1997) and their corresponding mean return periods are tabulated in Table 4. By referring to Table 4, there are two limit states in designing a precast double-storey building under performance levels namely, life safety limit state and collapse prevention limit state. Under life safety limit state, the probability of occurring earthquake within 50 years is 10% and the return period is 500 years. For example, life safety limit state for these buildings which designed under Basic Design Earthquake (DBE) for Wellington, New Zealand is taken as $F_vS_1 = 0.4g$ and for Malaysia is $F_vS_1 = 0.12g$ (see Figure 1) where g is defined as peak ground acceleration for any particular area. The collapse prevention limit state is defined as 2% probability occurrence earthquake exceeding 50 years with mean return period of 2500 years. The limit peak ground acceleration for collapse prevention or also known as "Maximum Considered Earthquake" (MCE) is taken as the value of $F_vS_1 = 0.8g$ for Wellington, New Zealand and $F_vS_1 = 0.22g$ for Malaysia. However, the value for DBE and MCE is depending on the location of these buildings to the earthquake epicenter. The DBE and MCE which denoted as dotted line in fragility curve which can be utilized to predict the percentage of Confidence Interval for the performance levels such as operational, immediate occupancy, life safety and collapse prevention.

Table 4. Probabilistic Hazard levels

Performance Level	Earthquake Having Probability of Exceedance	Mean Return Period (years)
Operational	50%/50year	75
Immediate Occupancy	20%/50year	225
Life Safety	10%/50year	500
Collapse Prevention	2%/50year	2500

In order to plot fragility curve for double-storey precast house, the theoretical equations together with design earthquake levels of DBE and MCE need to derive first. The first step of developing fragility curve is to set the spectral acceleration amplitude of an earthquake for a period of T = 1 sec and then, the drift damage limit must be converted into spectral acceleration units (A). Base shear demand (C_d) for period of the structures for high damping is given in equation (1) as stated by FEMA 273 (1997)

$$C_d = \frac{SA}{TB_L} \tag{1}$$

in which S is soil type factor, A is the peak ground acceleration (normalized with respect to g), T is the period of vibration and B_L is the factor of damping which is taken as more than 5%. The second step is to calculate the structural period of vibration according to yield strength and displacement for Single Degree of Freedom (SDOF) as given in equation (2).

$$T = 2\pi\sqrt{\frac{m}{K}} = 2\pi\sqrt{\frac{W\Delta}{Fg}} = 2\pi\sqrt{\frac{\Delta}{C_c g}} \tag{2}$$

where the base shear capacity of the structure is defined as $C_c = \frac{F}{W}$, where F is the yield strength (base shear) of the structure, W is seismic weight of the structures, Δ is the yield displacement of the structure, and K is the stiffness of the structures. By substituting equation (2) into equation (1) and equating base shear capacity equal to base shear demand, the equation becomes:

$$C_c^{\,2} = C_d^{\,2} = \frac{C_c g}{4\pi^2\Delta}\left(\frac{SA}{B_L}\right)^2 \tag{3}$$

Then, by substituting $\Delta = \theta H$, then

$$(SA)_i = 2\pi B_L\sqrt{\frac{C_c \theta H}{g}} \tag{4}$$

The third step is to convert the damage drift limit to spectral acceleration. Equation (4) is used to convert from damage drift limit to the spectral acceleration in developing the fragility curves. The fourth step is to transform the spectral acceleration into cumulative probability function (CPF). According to Mander (2003) the items which should be considered in developing fragility curves by taking into account the theoretical cumulative probabilistic functions are as follows:

(i) Expected site-specific response characteristics;
(ii) Inelastic strength and deformation capacity of the structure;

(iii) Damage limit states;
(iv) Randomness of ground motion response spectral demand;
(v) Uncertainties in modeling structural capacity.

The intersection of the capacity curve and appropriate damped elastic demand curve provides a "performance point" based on the estimate of the structural strength and displacement demand. The probability distributions over these two curves indicated the uncertainty and randomness of the structures performance with a wide range of possible performance outcomes. The randomness and uncertainty can be represented as probability distribution function. This distribution function can be expressed as a lognormal cumulative probability density function known as "fragility curve". The cumulative probability function is give by equation (5) as

$$F(SA) = \Phi\left[\frac{1}{\beta_{C/D}} \ln\left(\frac{S_a}{A_i}\right)\right] \quad (5)$$

Where Φ = standard log-normal cumulative distribution function; S_a = the spectral amplitude (for a period of $T = 1\sec$); A_i = the median spectral acceleration necessary to cause the i[th] damage state to occur and $\beta_{C/D}$ = normalized composite log-normal standard deviation which incorporates aspects of uncertainty and randomness for both capacity and demand.

The fifth step is to use central limit theorem by incorporating the normalized composite log-normal standard deviation. The central limit theorem requires the composite performance outcome to be distributed log-normally. By using the derivation of this theorem, the coefficient of variation for lognormal distribution is given by Kennedy at el. (1980);

$$\beta_{C/D} = \sqrt{{\beta_C}^2 + {\beta_D}^2 + {\beta_U}^2} \quad (6)$$

The value of $\beta_C = 0.2$ represented as randomness of the structural capacity based on the analysis carried out by Dutta and Mander (1998). β_U = uncertainty associated with strength reduction factor and the global modeling process, the assumed values is ranging between 0.2 and 0.4. The overall value of $\beta_{C/D}$ is calibrated by Pekcanet al. (1999) and validated by Dutta and Mander, (1998) against fragility analysis based on the site data obtained in the 1994 Northridge Earthquake and the 1989 Loma Prieta Earthquakes which recommended the value to be $\beta_{C/D}$ =0.6. After obtaining all the parameters of the cumulative probability function, the final step is to plot fragility curve for double-storey house using precast wall panel using lognormal distribution function as explained in the following section.

6. Results and Discussion

The fragility curves are used to represent the probabilities that the structural damages, under various levels of seismic excitation, exceed specified damage states. Figure 6 shows the fragility curves for double-storey house together with seismic vulnerability assessment performance when classified under coloured-coded and damage states numbering format. These fragility curves plotted based on equation (5) and equation (6) as derived above. The x-axis represents the Peak Ground Acceleration (PGA) which denoted as F_vS_1 and both y-axes represent Cumulative Probability Function (CPF) and Confident Interval which measured in percentages. The percentage Confident Interval (CI) is taken as the value of one subtracted from the value of Cumulative Probability Function and multiplied by 100%.

For low seismic region as Malaysia, the Design Basic Earthquake (DBE) which refer to the probability of 10% occurrences within 50 years or mean return period of 500 years is PGA= 0.12g while Maximum Considered Earthquake (MCE) with probability of 2% occurrences within 50 years or mean return period of 2500 years with PGA = 0.22g. Under DBE with PGA= 0.12g, the percentage confidence interval level would be 40% under green colour-tag, and 95% percentage confidence interval under yellow tag. Green colour-coding refers as fully functional and yellow colour-coding refers as functional for these buildings. It can be concluded that this building is still below the life-safety requirement, survive under PGA=0.12g and safe to be occupied after the earthquake. Under MCE with PGA=0.22g, the percentage confidence interval for green colour-coding is 10% , 65% confidence interval for yellow colour-coding, 85% confidence interval for orange colour-coding and 95% confidence interval for red colour-coding. It can be summarized that this building experience a significant structural damages and worst condition is that it will experience partial collapse of the buildings at PGA=0.22g. Therefore, this building will not survive under Maximum Considered Earthquake. The worst scenario will occur if the PGA of DBE and MCE increase to PGA=0.4g and PGA=0.8g, respectively.

Finally, it is suggested that this building needs to be designed using current seismic code of practice such as Eurocode 8 in order to survive under Maximum Considered Earthquake by increasing the percentage of reinforcement bars concrete, improved the strength capacity of buildings, better connection at beam-column interfaces and wall-column interfaces and lastly, increase the ductility of the system by increase the percentage drift of the structure.

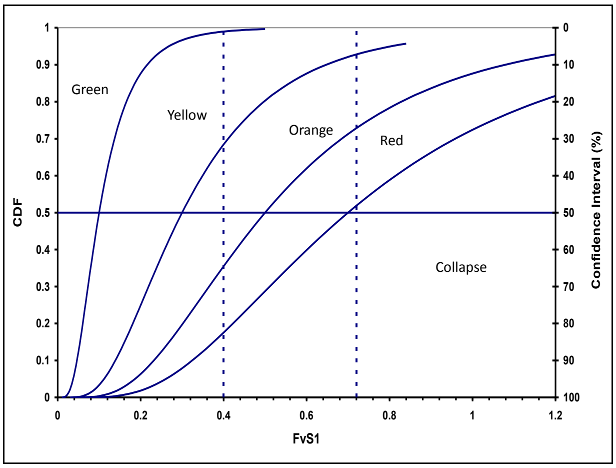


Figure 6. Fragility curve for precast double-storey house constructed using shear-key wall panel.

7. Conclusions and Recommendations

Based on the experimental results and discussion on precast double storey house, the following conclusions and recommendation are drawn:

1) Visual observation on the damages of precast shear-key wall are captured until 0.7% drift and the overall seismic performance of this type of building under reversible cyclic loading is poor and experience severe structural damage.
2) Precast double-storey house has 40% confidence interval of green colour-coding (fully functional) and 95% confidence interval of yellow colour-coding (functional) under Design Basic Earthquake (DBE) with PGA=0.12g.
3) Double-storey house has 10% confidence interval of green colour-coding (fully functional), 65% confidence interval of yellow colour-coding (functional), 85% confidence interval of orange colour-coding (life safety) and 95% confidence interval of red colour-coding (near collapse) under Maximum Considered Earthquake (MCE) with PGA=0.22g.
4) It is recommended that shear-key precast wall panel need to design in accordance to current seismic code of practice such as Eurocode 8 to cater for lateral seismic loading which comes from earthquake. Some modifications and improvements on the joints between wall-foundation interface, wall-column interface and wall-beam interface need be focused.

References

Banon et al. (1981).Seismic Damage in Reinforced Concrete Frames. Journal of Structural Division,ASCE, 107(9).pp 1713-1729.

Banon H, and Veneziano D(1982).Seismic Safety of Reinforced Concrete Members and Structures.Earthquake Engineering Structural Dynamics. Vol. 10, 1982.pp 179-193.

Blejwas B and Bresler B (1978).Damageability in Existing Buildings.EERC Rep. No.UCB 71-13. University ofCalifornia, Berkeley, 1971 UCLA, March 1978.pp.505-512.

Di Pasquale E, and Cakmak AS(1990).Seismic Damage Assessment using Linear Models.Soil Dynamic and Earthquake Engineering, 9(4).pp 194-215.

Dutta A, and Mander JB(1998).Capacity design and fatigue analysis of confined concrete columns. Technical Report.MCEER-98-0007,Multidisciplinary Center for Earthquake Engineering, State University of New York, Buffalo, NY, U.S.A

FEMA 273:(1997). NERHP Guidelines for the Seismic Rehabilition of Buildings and FEMA 274:(1997) "Commentary". Federal Emergency Management Agency, Washington, U.S.A.

HAZUS 99-SR2(2004).Earthquake Loss Estimation Methodology.HAZUS Technical Report, Federal Emergency Agency and National Institute of Buildings Science, Washington D.C. http://www.fema.gov/hazus/[accessed 4/4/10).

Ichii K (2004).Fragility Curves for Gravity-Type Quay Walls based on Effective Stress Analyses.13th World Conference on Earthquake Engineering. Vancouver, B.C., Canada, August 1-6, 2004.paper No.3040.

Kennedy et al., (1980).Probabilistic Seismic Safety Study of an Existing Nuclear Power Plant.Nuclear Engineering and Design, No.59.pp5-338.

Kircher et al., (1997), Development of Building Damage Functions for Earthquake Loss Estimation.Earthquake Spectra. EERI, November 1997, Vol. 13, N.4.pp663-683.

Mander JB(2003).Beyond Ductility: The Quest Goes On.Symposium to Celebrate the Lifetime Contributions of Professors Emeriti Tom Paulay and Bob Park. Christchurch, New Zealand.pp75-86.

O'Rourke MJ, and So P (2000).Seismic Fragility Curves For On-Grade Steel Tanks. Earthquake Spectra, Volume 16, No.4(November 2000).pp801-815.

Park YJ, and Ang AS (1985).Seismic Damage Analysis of Reinforced Concrete Buildings.Journal of Structural Engineering. ASCE, Vol. III, No. 4.pp740-757.

Pekcan et al., (1999).Fundamental Considerations for the Design of Non-Linear Viscous Dampers. Earthquake Engineering and Structural Dynamics, N28. pp1405-1425.

Singhal A, and Kiremidjan AS (1996).Method for Probabilistic Evaluation of Seismic Structural Damage.Journal of Structural Engineering, ASCE,Vol 122, No.12(December, 1996).pp 1459-1467.